Montana Department of Transportation
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Memorandum

To: Devin Roberts, P.E.
Bridge Area Engineer-Glendive

From: Jeff Jackson, P.E.
District Geotechnical Engineer - Glendive

Thru: William S. Warfield, V, P.G.
District Geotechnical Manager
Glendive District

Date: December 11, 2006

Subject: Geotechnical Engineering Structure Report (466)
Redstone E & W and JCT S-374-West
STPP 22-1(5)15
UPN 02024 and 02024001

The geotechnical section has been requested to provide a geotechnical engineering report for the proposed structures over Redstone Creek, Eagle Creek, and Big Muddy Creek on the Subject Projects. This report includes the results of the subsurface exploration, laboratory tests, analyses, and geotechnical recommendations in relation to the design of the bridge foundations. Geotechnical recommendations for the design and construction of the project alignment and minor structure features were provided in a Geotechnical Alignment Report dated June 27, 2006. These projects were originally designed as a single project that was split for funding purposes. Since our field investigation, laboratory testing, and initial analyses were initiated under one project we are provided a single report to address both projects.

Project Location and Information

These projects are located on Montana Highway 5 in Daniels and Sheridan Counties. The projects begin approximately 11.8 km west of Redstone at approximate Reference Post (RP) 14.8 (Station 236+50) and extend approximately 24.8 km (15.4 miles) easterly to Station 484+36.11. The projects proceed through rolling terrain used primarily for dryland farming and grazing and through the Big Muddy Creek Valley.

The intent of both projects is to reconstruct the roadway to a 9.2 meter finished top width. The replacement of numerous existing drainage structures and three bridges will also be accomplished under the projects.

The proposed alignment will be offset from the Present Traveled Way (PTW) in many locations, but at select areas will utilize the PTW within the new embankment prism. The proposed alignment will contain significant changes to the existing horizontal and vertical alignments. The projects include major grading, gravel, plant mix surfacing, bridge replacements, drainage structures, signing, and pavement markings.

Replacement bridges will be constructed at Eagle Creek, Big Muddy Creek, and Redstone Creek. We understand that the existing PTW and bridges will be used to maintain traffic while the new structures are constructed.

Area Geology

Geologically, the section from approximate Stations 236+50 to 316+00 is located in the Tertiary Flaxville Formation. This formation is composed of gravel, clay, and sand with local marl and volcanic ash. The gravel portion of the formation is composed primarily of quartzite clasts with numerous other rock types such as chert, agate, and quartz represented. The Redstone Hill portion of the roadway descends through part of the Tertiary Fort Union Formation. This formation is composed of yellow to buff sandstone, buff sandy shale, silty and carbonaceous shales, and thin beds of impure coal. The formation weathers to fine-grained sands, silts, and clays. From approximate Station 326+00 and extending east geology primarily consists of Quaternary alluvial deposits of the Big Muddy Creek valley. These soils are composed predominately of clay, silty clay, and fine sand with occasional gravel clasts and lenses.

Subsurface Investigation

The MDT Field Investigation Unit advanced eight borings at the proposed bridge locations from August 2005 through June 2006, these borings were advanced in addition to approximately 60 borings that were advanced as part of the investigation for the roadway reconstruction.

The borings were advanced with an all terrain Central Mine Equipment (CME) 1050 or 850 drill rig. Drilling was performed utilizing hollow stem augers and a casing advancer with water as a drilling fluid. The casing advancer was primarily used at depths below approximately 5 meters (15 feet) where heaving sands were problematic when drilling with augers. Subsurface sampling procedures included the Standard Penetration Test (SPT), and obtaining relatively undisturbed samples by hydraulically pushing Shelby Tubes. Samples were obtained in accordance with generally accepted geotechnical procedures.

Upon completion of our initial drilling program in February 2006, the horizontal alignment near Eagle Creek was moved to the north, thus additional subsurface investigations (Borings 68 and 69) were performed at the revised Eagle Creek bridge location.

Advancement of the borings were generally observed and logged by a representative of the MDT Geotechnical Section. District construction personnel surveyed the boring locations and provided the State Plane coordinates and ground surface elevation to the Geotechnical Section for each boring.

Subsurface Conditions

Subsurface soil conditions at all three bridge locations generally consist of interbedded very soft to soft clay or silt and loose to very loose clayey sand extending in depths to approximately 20 meters (65 ft.) and in some borings deeper. Standard Penetration Test results (N values) were consistently below five in these upper reaches and at numerous boring and sample locations, the sampler penetrated the subsurface soils under the weight of the drilling rods or the weight of the hammer and dynamic impact from the hammer was not required to advance the sampler.

At depths below about 20 meters (65 ft.) the clay and sand soils generally become slightly stiffer or denser, although blow counts were still relatively low. Medium dense to dense gravel layers were encountered sporadically at depths below 20 meters (65 ft) within some of the boring locations but were sometimes not encountered until depths of 40 meters (130ft). Formation material was not encountered in any of the bridge borings some of which were drilled to depths of 44 meters (145 feet).

In general, the majority of the subsurface soils encountered at all three bridge locations are considered soft, compressible, and exhibit low shear strengths. Generally, dense gravel or formation material is not present in the upper 30 meters (100 ft) of the subsurface. Based upon these soils, deep piles obtaining most of their capacity through friction will be needed to support the superstructure loading.

More detailed subsurface soil information is provided on the attached boring logs.

Bridge Foundation Design and Construction Recommendations

The soils extending down to approximately 17 to 20 meters (55 to 65 ft) at all three bridge locations are highly compressible when subject to loading from the approach embankments. The amount of settlement expected to occur based upon the proposed fill height, ground water elevations, and corresponding stress distribution at depth, will be large enough to mobilize negative skin friction along the piles to depths of approximately 20 meters (65 ft). At depths below about 20 meters (65 ft) the stress distribution decreases to a point where the predicted settlement will be essentially negligible.

Negative skin friction (also called negative shaft resistance or drag load) will increase the loading on the piles as the subsurface soils settle in relation to the pile. This settlement results in increased loading on the pile, thus resulting in larger piles, longer piles, and higher required ultimate capacities during driving. Our calculations indicate the potential down drag loading at some bent locations on this project approaches and even slightly exceeds the magnitude of loading applied to the piles from the bridge superstructure itself. The potential for these magnitudes of loading induced from negative skin friction can be greatly reduced by constructing the approach embankments and allowing the subsurface soils to settle under the embankment loads before the piles are driven.

The compressible clay layers are highly interbedded with seams of sand and the relative thicknesses of the clay layers are not excessive. Thus, the majority of settlement within the clay layers is expected to occur within a few months time after the embankments are constructed, and the elastic settlement within the sand layers is expected to occur during embankment construction. Based upon our consolidation lab tests and analyses, we estimate that by allowing the approach embankments to settle for a minimum of 45 days, the potential for negative skin friction will be greatly reduced, and the resulting ultimate capacities required during driving will also be reduced.

To reduce the potential negative skin friction, we recommend constructing the approach embankments prior to driving the piles at all three bridge locations and the design recommendations contained within the rest of this report assume that the approach embankments will be constructed, the subsurface soils allowed to settle for a minimum of 45 days, and the piles will be driven through the approach embankments. Based upon the offset alignment and size of this project, we do not anticipate that the required waiting period would drastically alter the Contractors schedule.

In the event the approach embankments cannot be constructed prior to pile driving for unforeseen reasons at this time, piles driven to greater depths and higher ultimate capacities during driving will be required to resist the down drag loads, larger diameter piles will also be required at some bent locations. The Geotechnical Section should be notified to re-assess our recommendations in the event the approach embankments can not be constructed and subsurface soils allowed to settle prior to pile driving.

<u>Seismicity and Liquefaction</u>
Although the probability of strong ground motion to occur in eastern Montana from a seismic event is considered low, there is a slight increased risk in far northeastern Montana. USGS published information indicates seismic events occurred in 1909 and 1943 with estimated ground shaking intensities of V to VI based upon the Modified Mercalli Scale. The 1909 earthquake had an estimated Richter magnitude on the order of 5.5 and was felt as far west as Helena. Minor structural damage to residences within Redstone and the surrounding communities were reported after the seismic event in 1943. Although the return intervals for strong ground motions are low in this area, the possibility for low to moderately strong ground motion does exist within this project area. Published literature correlates Modified Mercalli intensities of V to VI to be "roughly" equivalent to peak ground accelerations on the order of 0.05g to 0.2g.

Recently performed work prepared for the DNRC Dam Safety Program by Wong et al (MBMG Special Publication 117) includes ground shaking maps for the state of Montana based upon probabilistic hazard analyses.

Ground motion prediction and modifications of existing and development of new attenuation relationships to estimate peak ground accelerations is a rapidly changing field within geotechnical engineering and we believe the peak ground accelerations developed as part of this recent work to be the most updated research to our knowledge.

Therefore, the ground accelerations determined from this recent work and the USGS published data (1996 and 2002) have been used to assess liquefaction susceptibility for the subsurface soils in this area. Based upon both the recent work by Wong et al and the USGS data, a peak ground acceleration (PGA) on rock of approximately 0.1g was used for both the 10% and 2% exceedence in 50 yr events (approximate 500 year and 2500 year return intervals, respectively). The same value of peak ground acceleration is reported in the MBMG publication for both return intervals in eastern Montana due to the low probability of seismic activity and relatively long distances from active known faults. For comparison, The PGA on rock values from the 2002 USGS mapping indicates approximately 0.03g and 0.12g for the 10% and 2% exceedance in 50 year events, respectively.

The MBMG publication does indicate (and is commonly known) that ground motions are generally amplified near rivers, streams, and valleys where deep fluvial or lacustrine deposits exist and ground accelerations could be amplified to values on the order of 0.2g near the ground surface in these areas. These types of deep deposits are present at all three bridge locations on this project as identified during our subsurface investigation program.

Preliminary liquefaction analyses were performed based upon the simplified Youd and Idriss procedure published in 2001; this procedure is an updated version of the commonly used Seed & Idriss method and is generally considered state of practice for preliminary liquefaction assessments. Based upon the results of our analysis, the subsurface soils are not likely to liquefy when subject to an acceleration on the order of 0.1g, however liquefaction of select sand layers is expected to occur for ground accelerations on the order of 0.2g or greater.

Most of the subsurface sands at the bridges contain a significant percentage of fines, and thus are much less susceptible to liquefaction. However, there are isolated thin sand layers within the subsurface profile where the fines are less prevalent. These soil layers are very loose and saturated, and are thus susceptible to liquefy when subject to ground accelerations at or above 0.2g. The liquefiable layers are generally located within the upper approximate 21 meters (70 ft.) of the soil stratum, and above the design pile tip elevations. We anticipate that the outcome from a seismic event strong enough to induce liquefaction will likely be minor settlement of the subsurface soils and potentially minor damage related to the subsurface soil settlement.

Mitigation to reduce the likelihood of liquefaction will be extremely expensive and would necessitate ground improvements to densify the liquefiable soils at depth. Although our analyses indicate the likelihood of liquefaction is relatively low, there have been documented seismic events in this area that could possibly produce strong enough ground motion to induce liquefaction and we believe it prudent to address the potential for liquefaction within this report. The Geotechnical Section will perform a more in-depth liquefaction assessment and corresponding estimate of mitigation costs at the request of the Bridge Bureau (or others) in the event the Department elects to design the proposed bridges to potentially higher ground accelerations on the order of 0.2g, which would be beyond the current AASHTO standards as we understand them.

Bridge Foundation Recommendations

Eagle Creek

Loading information:
The Eagle Creek bridge is anticipated to be a two span, type IV prestressed concrete structure with five piles per bent. Preliminary loads were provided to the Geotechnical Section by the Bridge Bureau via memorandum on July 31, 2006. The preliminary loads in Table 1 were provided for the Eagle Creek crossing and determined using the AASHTO LRFD Bridge Design Specifications.

Table 1

Abutment Axial Load (kN/pile)	Pier Axial Load (kN/pile)	Axial Load Combination	Lateral Load -Piers Only- (kN)	Elevation of Applied Lateral Force (m)	Lateral Load Combination
874	1428	Service	416	643.43	Extreme Event

Bridge Support:
A steel pile system consisting of 610 X 19.05 mm closed end piles are recommended at the intermediate bent and 508 x 12.7 mm closed end piles are recommended at the end bents. The larger piles at the intermediate bent are required to resist both the lateral loading and support the significantly higher axial loading. We recommended that steel piles with minimum yield strength of 310 MPa be used and that flat plates be welded to the pile ends for closure. See Table 2 for design tip elevations and required ultimate capacities during driving.

Pile tip elevations and ultimate capacities during driving for bents 1 and 3 in Table 2 were calculated assuming the approach embankments are constructed and allowed to settle for a minimum of 45 days before the piles are driven at bents 1 and 3. The Minimum Required Axial Ultimate Capacity During Driving listed in this table assumes that the piles will be driven through the approach embankment fill. In addition to the loading generated by the service loads, the piles will need to support the remaining drag load generated by the settlement under the approach embankment layer after the 45 day waiting period. Therefore, the ultimate capacities required during pile driving at the end bents are greater than those required by the service loads including standard factors of safety.

Table 2

Bent	Pile Type and Size	Approximate Bottom of Pile Cap Elevation (m)	Design Pile Tip Elevation (m)	Minimum Required Axial Ultimate Capacity During Driving (kN)
1	PP 508X12.7 mm	642.6	610.1	3850
2	PP 610X19.05 mm	642.2	604.3	4100
3	PP 508X12.7 mm	642.7	607.9	3800

**If the piles have not achieved the required minimum ultimate capacity during driving at the design tip elevation, the piles should be driven deeper as directed by the Geotechnical Engineer.*

The required design pile tip elevation and required minimum ultimate pile capacities should be shown on the drawings for each bent along with the above note. Pile spacing should be a minimum of 3 pile diameters center to center. The standard note indicating a 24-72 hour time window for restrike will be appropriate.

Lateral deflection at the interior bent is estimated to be approximately 80 mm at the top of the pile. L-pile files were transmitted to the Bridge Bureau via e-mail on December 8, 2006.

Pile tip settlement is estimated to be less than 25 mm at all bents assuming the approach embankments are constructed and allowed to settle before piles are driven at bents 1 and 3.

A wave equation analysis will be performed by the geotechnical section for approval of the contractor's proposed pile hammer. A preliminary driveability analysis was performed using a common manufactures recommended driving system, these analyses indicate a pile hammer that produces a minimum energy of 150 KJ (111 kip-ft) will be required to drive the piles to the ultimate capacities required. During construction, one test pile should be driven at each bent location using a pile driving analyzer (PDA) and re-struck after a period of at least 24 hours prior to driving the production piles. The software program CAPWAP should be used to evaluate the PDA results. The Geotechnical Section will use the PDA results to select the required resistance criteria for the production piles. We recommend including a note in the plans indicating driving of production piles should not be initiated until completion of PDA testing and analysis of the PDA data is complete. Use of the PDA is covered in Section 559 of the Standard Specifications.

<u>Redstone Creek</u>

Loading Information:
The proposed Redstone Creek bridge will consist of a single span, Type 4 stub abutments with five piles per bent. Preliminary loads provided by the Bridge Bureau for Redstone Creek are provided in Table 3.

Table 3

Abutment Axial Load (kN/pile)	Axial Load Combination
844	Service

Bridge Support:
A steel pile system consisting of closed end 508 x 12.7 mm pipe piles is recommended for both bent 1 and 2. We recommended that steel piles with minimum yield strength of 310 MPa be used and that flat plates be welded to the pile ends for end closure. Table 4 depicts the design tip elevations and required ultimate capacities during driving.

Pile tip elevations and ultimate capacities in Table 4 were calculated assuming the approach embankments are constructed and allowed to settle for a minimum of 45 days before the piles are driven. The Minimum Required Axial Ultimate Capacity During Driving listed in Table 4 assumes that the piles will be driven through the approach embankment fill. In addition to the loading generated by the service loads, the piles will need to support the remaining drag load generated by the settlement under the approach embankment layer after the 45 day waiting period. Therefore, the ultimate capacities required during pile driving at the end bents are greater than those required by the service loads including standard factors of safety.

Table 4

Bent	Pile Type and Size	Approximate Bottom of Pile Cap Elevation (m)	Design Pile Tip Elevation (m)	Minimum Required Axial Ultimate Capacity During Driving (kN)
1	PP 508x12.7 mm	637.1	598.1	3560
2	PP 508X12.7 mm	637.1	596.3	3450

**If the piles have not achieved the required minimum ultimate capacity during driving at the design tip elevation, the piles should be driven deeper as directed by the Geotechnical Engineer.*

The required design pile tip elevation and required minimum ultimate pile capacities should be shown on the drawings for each bent along with the above note. Pile spacing should be a minimum of 3 pile diameters center to center. The standard note indicating a 24-72 hour time window for restrike will be appropriate.

A wave equation analysis will be performed by the geotechnical section for approval of the contractor's proposed pile hammer. Preliminary driving analyses were performed assuming the use of a manufactures recommended driving system, these analyses indicate a hammer that produces a minimum energy of 82 KJ (61 kip-ft) will be required to drive the piles to ultimate capacities required. During construction, one test pile should be driven at each abutment location (Bent No. 1 and Bent No. 2) using a pile driving analyzer (PDA) and re-struck after a period of at least 24 hours. The software program CAPWAP should be used to evaluate the PDA results. We recommend including a note in the plans indicating driving of production piles should not be initiated until completion of PDA testing and analysis of the PDA data is complete. The Geotechnical Section will use the PDA results to select the required resistance criteria for the production piles. Use of the PDA is covered in Section 559 of the Standard Specifications.

Big Muddy Creek

Loading Information:

The Big Muddy Creek bridge is a proposed 3 span, type MT-28 stub abutment structure with six piles per bent. Preliminary loads (per pile) provided by the Bridge Bureau for this bridge are depicted in Table 5.

Table 5

Abutment Axial Load (kN/pile)	Pier Axial Load (kN/pile)	Axial Load Combination	Lateral Load -Piers Only- (kN)	Elevation of Applied Lateral Force (m)	Lateral Load Combination
503	711	Service	289	636.32	Extreme Event

Bridge Support:

A steel pile system consisting of 610 x 19.05 mm pipe piles are recommended for the interior bents and 508 x 12.7 mm pipe piles are recommended for the end bents. It is recommended that steel piles with minimum yield strength of 310 MPa be used and that flat plates be welded to the pile ends for end closure. The larger piles at the interior bents are required to resist the lateral loading, also taking into account the potential scour.

Pile tip elevations and ultimate capacities for bents 1 and 4 in Table 6 were calculated assuming the approach embankments are constructed and allowed to settle for 45 days before the piles are driven at bents 1 and 4. The Minimum Required Axial Ultimate Capacity During Driving listed in Table 6 assumes that the piles will be driven through the approach embankment fill.

Table 6

Bent	Pile Type and Size	Approximate Bottom of Pile Cap Elevation (m)	Design Pile Tip Elevation (m)	Minimum Required Axial Ultimate Capacity During Driving (kN)
1	PP 508X12.7 mm	635.7	604.3	2875
2	PP 610X19.05 mm	635.7	607.1	3300
3	PP 610X19.05 mm	635.7	605.5	3350
4	PP 508x12.7mm	635.7	604.9	2900

**If the piles have not achieved the required minimum ultimate capacity during driving at the design tip elevation, the piles should be driven deeper as directed by the Geotechnical Engineer.*

The required design pile tip elevation and required minimum ultimate pile capacities should be shown on the drawings for each bent along with the above note. Pile spacing should be a minimum of 3 pile diameters center to center. The standard note indicating a 24-72 hour time window for restrike will be appropriate.

Lateral deflection at the top of the pile is estimated to be on the order of 60 to 75 mm at the interior bents for the extreme event lateral loading. L-Pile files were transmitted to the Bridge Bureau via e-mail on December 8, 2006. The lateral analyses were based upon scour occurring to an approximate elevation of 631.5m at bent 2 and 632.0m at bent 3, which corresponds to predicted scour levels with the proposed sheet pile installed.

Lateral deflections were also analyzed assuming that scour occurs to an approximate elevation of 630.0m at bent 3 (no sheet pile system installed), for which case the lateral deflections are estimated to be on the order of 125 mm at the top of the pile. This analysis was not performed at bent 2, as the predicted scour without sheet pile is less at this location.

A wave equation analysis will be performed by the geotechnical section for approval of the contractor's proposed pile hammer. Preliminary driving analyses were performed assuming the use of a manufactures recommended driving system, these analyses indicate a hammer that produces a minimum energy of 150 KJ (111 kip-ft) will be required to drive the larger piles to the ultimate capacities required. During construction, one test pile should be driven at each bent location using a pile driving analyzer (PDA) and re-struck after a period of at least 24 hours. The software program CAPWAP should be used to evaluate the PDA results. The Geotechnical Section will use the PDA results to select the required resistance criteria for the production piles. We recommend including a note in the plans indicating driving of production piles should not be initiated until completion of PDA testing and analysis of the PDA data is complete. Use of the PDA is covered in Section 559 of the Standard Specifications.

Sheet Pile at Big Muddy Creek Bridge

Based upon the memorandum from the Hydraulics Section dated March 3, 2006 we understand sheet pile has been proposed at the Big Muddy Creek Bridge to reduce the potential scour. Preliminary design indicates a top sheet pile elevation of 631.24m and minimum tip elevation of 620.34m. We have estimated a scour elevation of 630.4m on the stream side of the sheet piles corresponding to the predicted contraction scour of 0.84 meters for the overtopping event.

We have performed stability analyses for the sheet pile assuming 0.84 meters of unsupported height resulting from the predicted contraction scour. This unsupported height will need to retain the bank side soil and rip rap proposed on a 3H:1V slope. We recommend PZ27 sheet piles installed to the required minimum tip elevation of 620.34m. Based upon this embedment depth and pile size we estimate a deflection of approximately 60 mm at the top of the wall for this value of unsupported height and a factor of safety against overturning at the toe of approximately 2.0. For larger contraction scours resulting in higher unsupported lengths, larger and deeper sheet pile will be required.

Other Design and Construction Recommendations

We recommend select backfill at the bridge ends for all three bridges. A Special Provision, *Bridge End Backfill* has been attached to this memorandum. Positive drainage should also be provided at the base of the Bridge End Backfill.

Approach embankment end slopes are recommended to be 2H:1V or flatter and 3H:1V are preferred. We recommend using high survivability class C subsurface drainage geotextile (table 716-4 of the Standard Specifications) below the rip rap at all bridges. We may also have additional recommendations after our review of the preliminary bridge layout plans, once these plans become available.

We have recommended two different piles sizes for this project and the larger piles are required at the interior bents to resist the lateral loading. These larger piles could also be used at the end bents for all three bridges (if desired) so that one pile size could be used for all of the projects. The larger piles would not need to be driven to as great of depth as the smaller ones, but we suspect the slightly reduced lengths of the larger pile will be more than offset by the increased cost of the larger pile. Our recent experience also indicates that the larger piles with 19.05 mm (0.75 inch) wall thickness can be difficult to obtain on short notice. The Geotechnical Section can provide additional recommendations for the larger 610x19.05 mm pile at the end bents, if requested.

Based upon the subsurface soils present and geotechnical issues that have been identified for these projects the geotechnical section recommends a meeting to discuss the recommendations and various options that have been presented in both this report and primarily with our Activity 464 alignment report. We would suggest this meeting occur as part of the Plan-In-Hand meeting however, an earlier meeting could be scheduled if desired.

Questions regarding this memorandum or project may be directed to Jeff Jackson, MDT Geotechnical Section, 444-3371 or via email, jejackson@mt.gov.

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Attachments: Boring Logs
Boring Log Key
Special Provision for Bridge End Backfill
Special Provision for Sheet Pile

Original: Geotechnical Project File

Copies: Ray Mengel, District Administrator - Glendive
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Geotechnical Correspondence File